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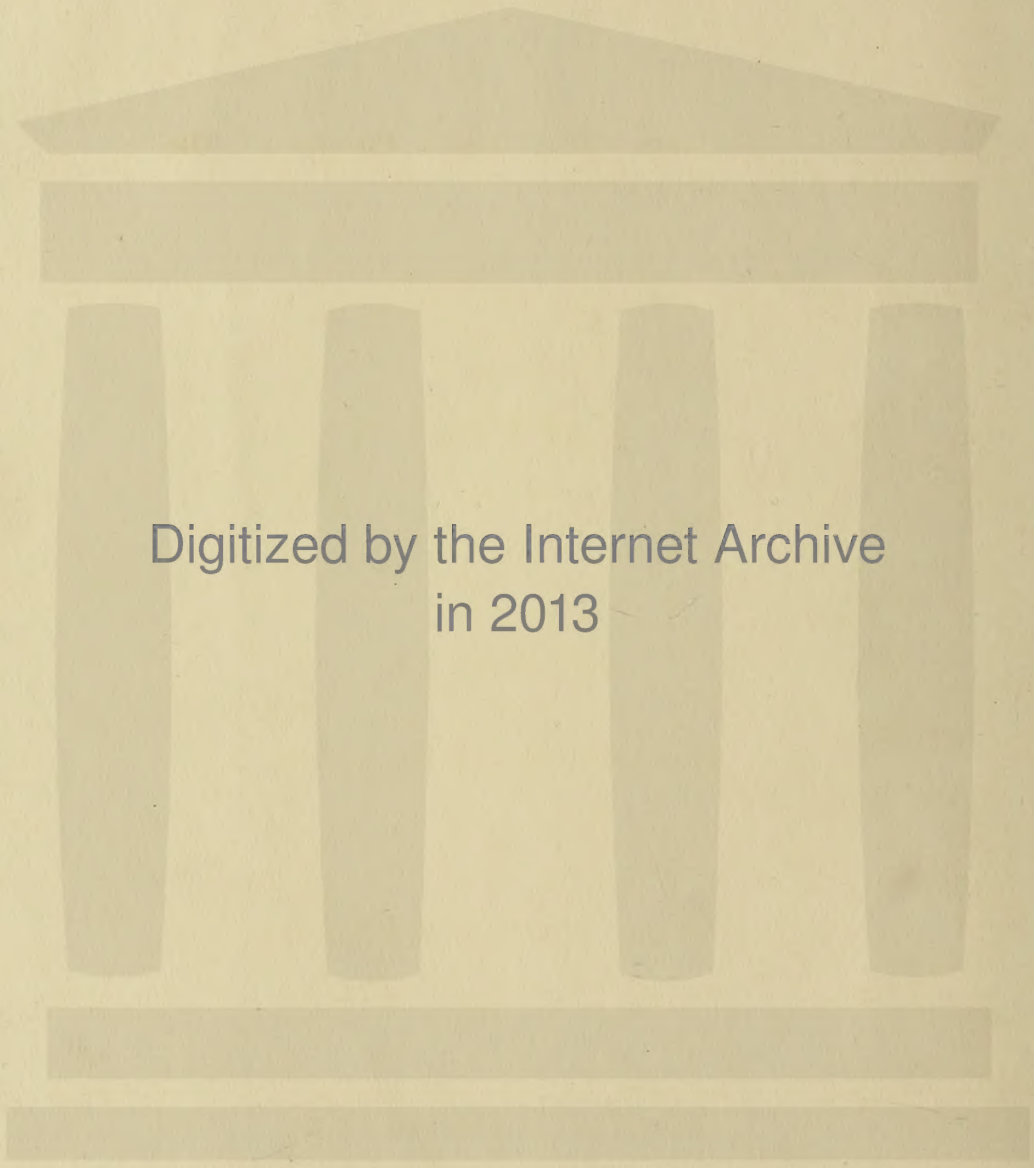
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INVESTIGATION OF A HIGHWAY BRIDGE

BY

THOMAS MERRITT PITTMAN JR

THESIS

FOR THE

DEGREE OF

BACHELOR OF SCIENCE

IN

CIVIL ENGINEERING

IN THE

COLLEGE OF ENGINEERING

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May 25, 1911

I recommend that the thesis prepared under my supervision by THOMAS MERRITT PITTMAN JR. entitled Investigation of a Highway Bridge be approved as fulfilling this part of the requirements for the degree of Bachelor of Science in Civil Engineering.

N. B. Garver
Instructor in Civil Engineering.

Recommendation approved:

Ira O. Baker.
Head of the Department of Civil Engineering.

197712

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Introduction.

For the purpose of becoming more familiar with the usage and interpretation of specifications, and of becoming better acquainted with bridge details, the investigation of an existing structure was chosen as the subject of this thesis.

The particular bridge investigated is a highway bridge which spans the Vermilion River at Danville, Illinois, and is locally known as the "Woolen Mill" bridge.

The following work is in the nature of a report on the bridge in regard to its present efficiency, together with recommendations based upon the investigation. The investigation was made in accordance with Theodore Cooper's revised specifications of 1909, for a bridge of A_2 class.

The dead load was computed from measurements of the bridge, and one third of it considered as acting at the panel points of the unloaded chord. The live load for the trusses was taken from the specifications, but the loading for the floor system was taken in accordance with local conditions as explained in Art. 6.

The efficiencies were computed by dividing the actual area of the section by the required area as determined from the stress and the allowable unit stress.

The writer wishes to express his thanks to Mr. N. B. Garver, under whom this thesis was prepared, for his valuable suggestions and criticisms.

Description of the Bridge

The "Woolen Mill" bridge is a highway bridge 716 ft. long, which spans the north fork of the Vermillion river, and connects Vermillion Heights with Danville, Illinois.

It consists of 19 twenties bents, a ten panel, deck Whipple truss, and five panel deck Pratt trusses. It was built about 1885 by the Lafayette Bridge Co. of Lafayette, Indiana.

The foundations are all either sandstone blocks or of rubble sandstone masonry, except the footings of the four most easterly columns, which are of concrete. The east end of the Whipple truss is supported by two capped steel cylinders, probably filled with concrete, which have sandstone footings. The remainder of the bridge is supported by latticed channel columns which, except as noted above, also rest on sandstone footings.

The roadway is 18 ft. wide, a four foot walkway on each side being separated from it by oak wheel guards. On the outside of each walk is a railing composed of angle posts and lattice webbing. The flooring and joists are of oak.

The bridge, for 200 ft. on the west end, and for 58 ft. on the east end, is on a 3% grade, the remainder being level. A profile sketch of the bridge is shown in Fig 1.

For convenience, the report will be

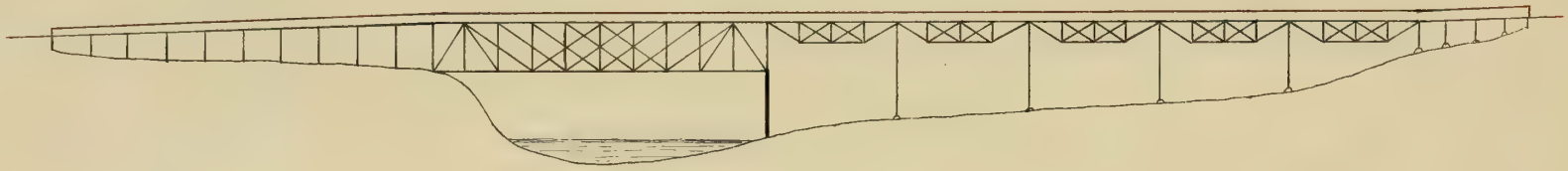


Fig. 1

"WOOLEN MILL" BRIDGE
DANVILLE ILL.

divided into six articles, which will in turn, consider the, Floor system, Whipple Truss, Pratt Trusses, Trestle Bents, Foundations, and Recommendations.

Art. 1. Floor System.

The Floor system of the bridge is the same for all sections, except in length of span. The longest span is between the trestle bents at the west end, and is 20 ft.

The floor system consists of oak planks 3 in. thick laid on 3 in. by 12 in. joists spaced 2 ft. center to center. The joists rest upon floor-beams composed of a web plate and four angles.

Floor-Beam

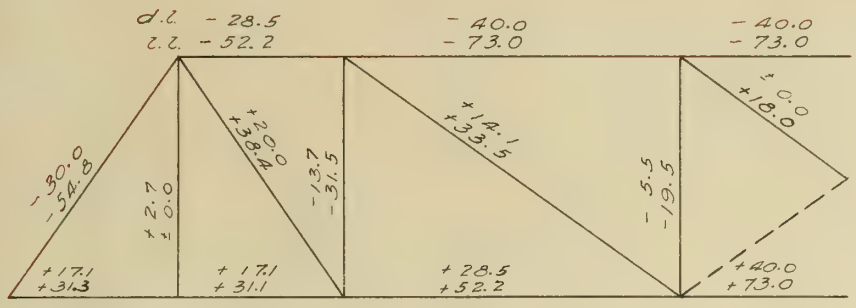
Fig. 2.

Assuming a uniform load of 100 lb. per sq. ft. over the entire roadway, the joists have an efficiency of 0.6, and the floor-beams have an efficiency of 0.83. Concentrated loads will not be considered for reasons stated in Art. 6

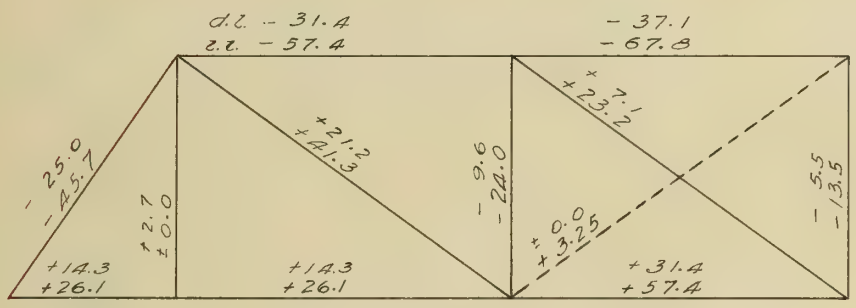
Art. 2. Whipple Truss.

The Whipple truss is a ten panel deck truss of 174 ft. span, which carries the roadway over the channel of the river.

The dead panel load, as computed from measurements, is 8100 lb., one third of which is assumed to act on the unloaded chord.



(a)



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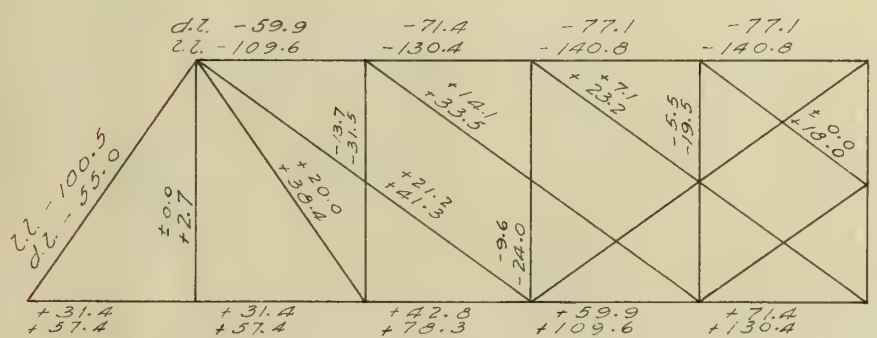


(c)

FIG. 3.
Calculation of Stresses
in
Whipple Truss

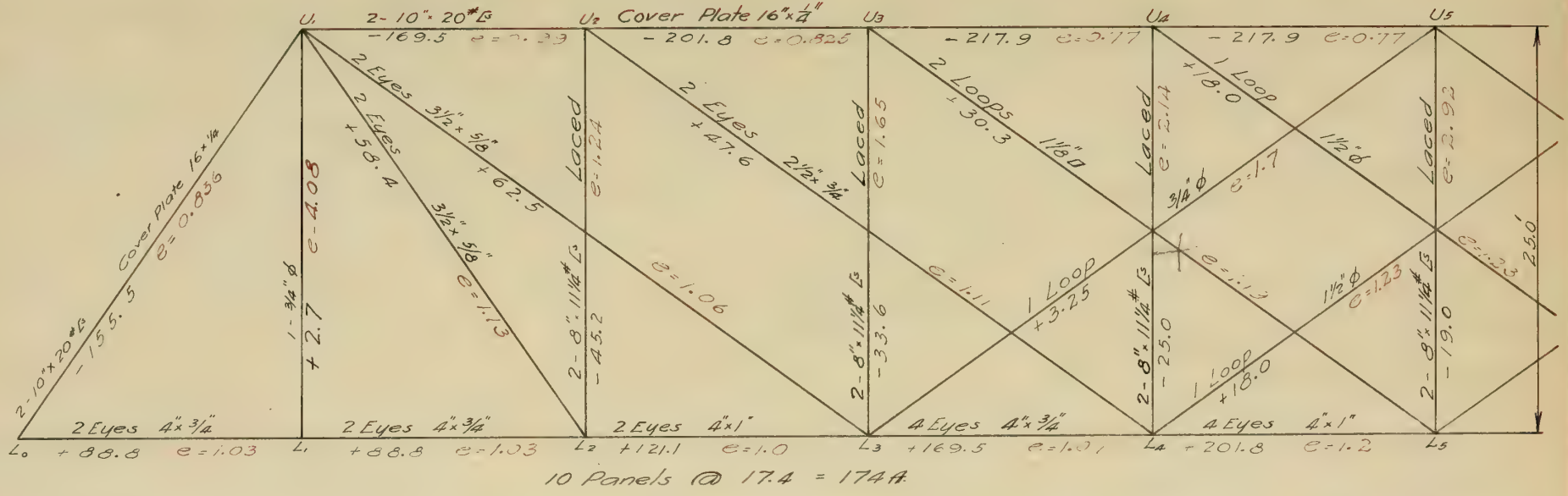
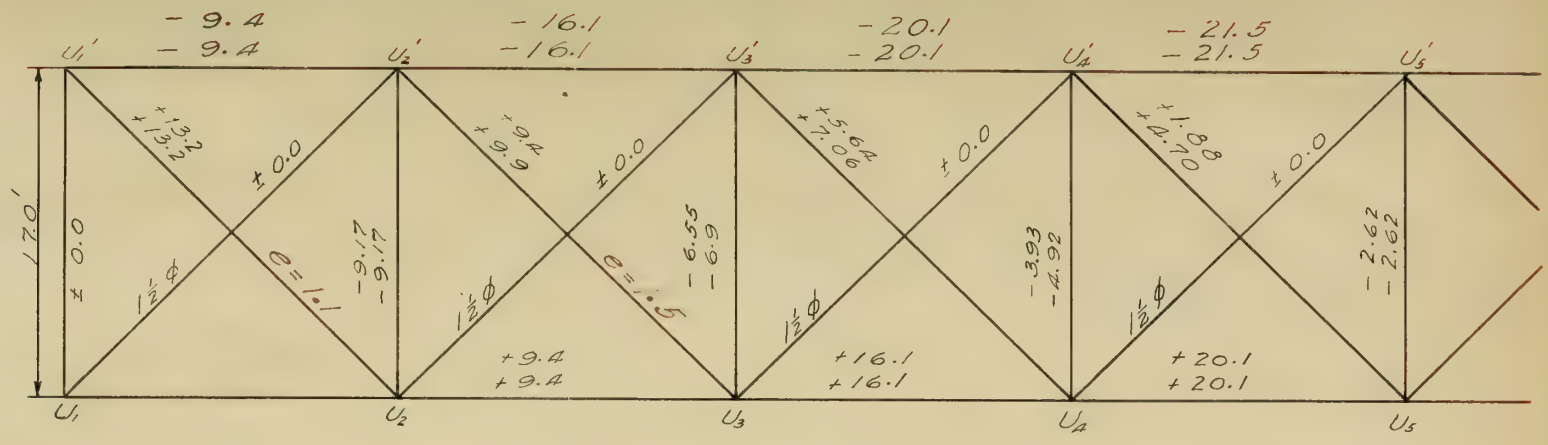


FIG. 4.

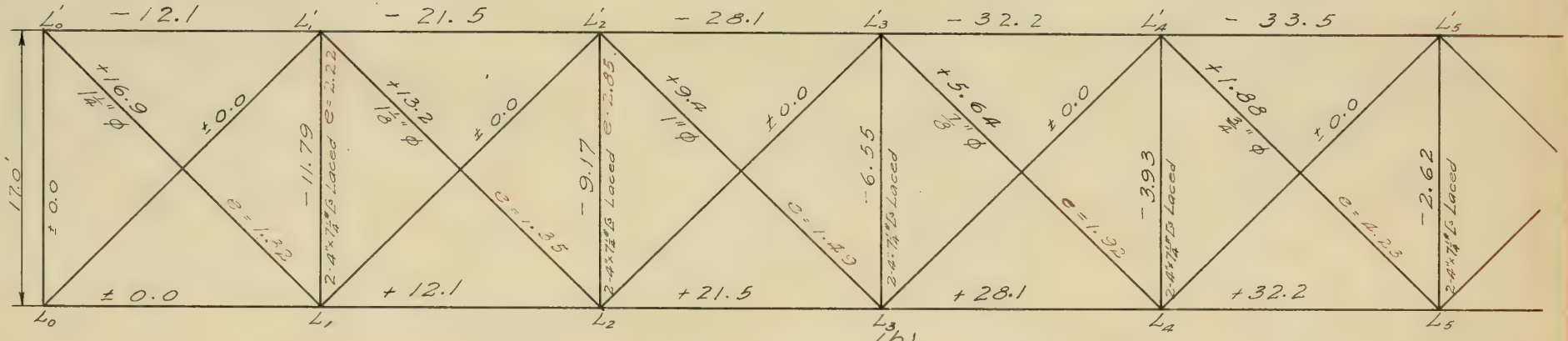
Whipple Truss

showing

Stresses, Sections and Efficiencies.



(a)
Top Lateral System
of
Whipple Truss.



(b)
Bottom Lateral System
of
Whipple Truss.

Fig. 5.

The live load, taken from the specifications Art. 38, is 65 lb. per sq. ft. which gives a live-panel load of 15000 lb., all of which acts on the upper chord.

The stresses due to wind were computed in accordance with the specifications, Art. 29.

Trusses: The composition of the trusses is shown in Fig 3, and the combined stresses, together with the sections of the members and the efficiency of each, are shown in Fig 4; the efficiencies being shown in red.

Lateral Systems: The stresses and the efficiencies of the members in the lateral systems are shown in Fig 5 (a) and (b).

Sway Bracing: The sway bracing in the plane of the end post consists of $1\frac{3}{4}$ in. bars which have a calculated efficiency of 2.29. The intermediate sway bracing consists of $\frac{7}{8}$ in. rods which have a calculated efficiency of 2.32.

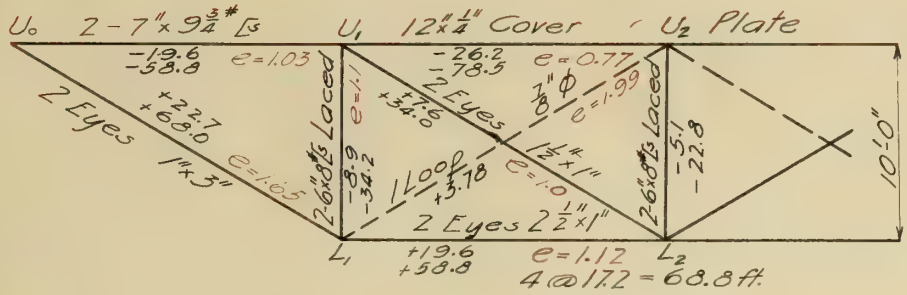
Art. 3. Pratt Trusses.

As shown in Fig. 1, there are five Pratt trusses. They are alike, so only one need be investigated.

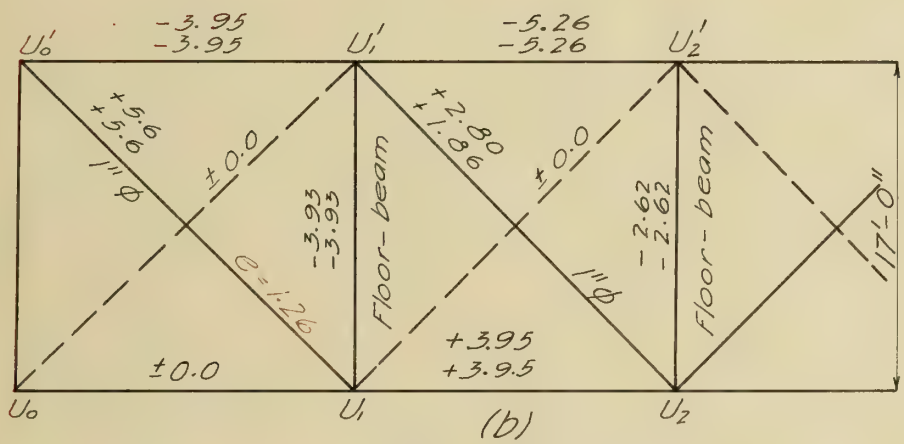
The Pratt truss is a four panel deck truss of 68.8 ft. span, which is supported on the steel bents on the east side of the river.

The dead panel load, as computed from measurements, is 7600 lb., one third of which is assumed to act on the unloaded chord.

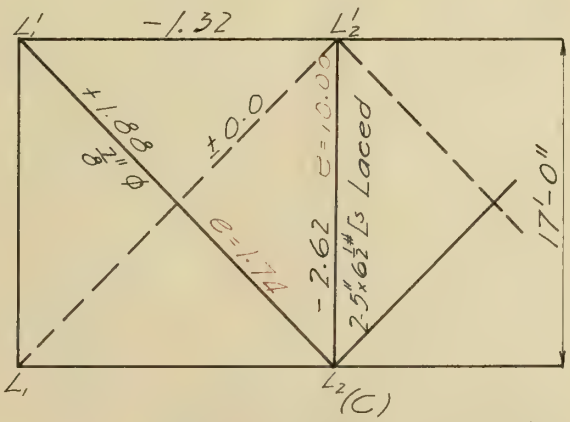
The live load, taken from the specifications Art. 38, is 100 lb. per sq. ft. which gives a live-panel load of 22800 lb., all of which acts on



(a)
Pratt Truss.



(b)
Top Lateral System
of
Pratt Truss.



(c)
Bottom Lateral System
of
Pratt Truss.

Fig. 6.

the upper chord.

The stresses due to wind were computed in accordance with the specifications Art. 39.

Trusses: The stresses in the members together with the sections and efficiencies of the members, are shown in Fig. 6. (d).

Lateral Systems: The stresses in the members of the top lateral system, together with the sections and efficiencies of the members, are shown in Fig. 6, (b), and those of the bottom lateral system are shown in Fig. 6, (c).

Sway Bracing: The sway bracing consists of $\frac{7}{8}$ in. rods. The intermediate bracing has a computed efficiency of 3.55 and that in the plane of the end post has a computed efficiency of 1.66.

Art. 4. Trestle Bents.

The trestle bents on the west end of the bridge are similar, so only the longest one will be considered here.

The bent consists of two latticed channel columns placed 17 ft. apart and braced in two directions.

The columns are braced longitudinally by two knee-braces connecting with a strut. The transverse bracing consists of two diagonal rods, each extending from the foot of one column to the top of the other. Fig. 7 shows the arrangement of the bracing.

In accordance

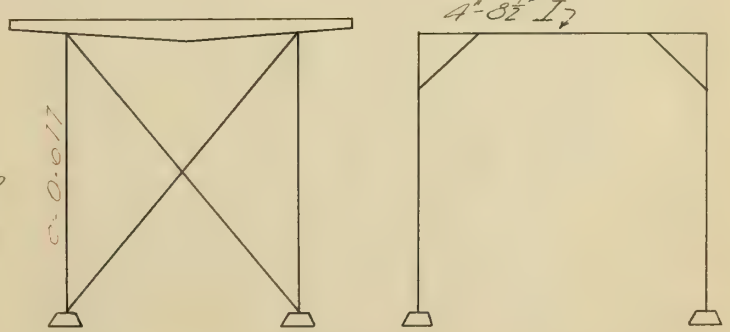


Fig. 7.

with the specifications and the preceding articles, the axial dead load is 7000 lb., the axial live load 27000 lb., and the horizontal wind load 3100 lb., 1050 lb. being applied at the top of the columns, and 1050 lb. being applied at the foot of the columns.

This loading gives an efficiency in the columns of 0.677.

The trestle bents on the east side of the river are higher than are those on the west side, and consist of two battered channel columns, which in addition to the floor-beam are braced transversely by three horizontal struts and the usual diagonal bracing. There is no longitudinal bracing whatever. The arrangement of the members, together with the specifications of each, is shown in Fig. 8.

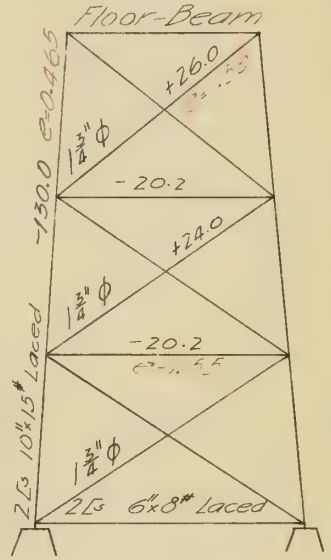


Fig. 8.
Trestle Bent on East End

Art. 5. Foundations.

As stated in the description of the bridge, all of the columns and each of the cylinders rest upon independent footings. With the exception of the four most easterly columns, the footings are of either sandstone "ocks" or rubble sandstone masonry. The footings of the first bent east of the river show slight signs of deterioration, but not enough to require consideration. The remaining footings

are in good condition and show no signs of deterioration. The footings above referred to as concrete footings are very probably footings, similar to the others in use, which have been covered with a veneer of concrete. They appear to be in good condition. Most of the foundations are on high ground and all, except the footings of the cylinders, are surrounded by grass, weeds, and low underbrush which protects them from scouring during times of high water. In fact there is no evidence at all of scour having taken place at any of the foundations. The west abutment is in good condition, but the east abutment, though not dangerous, is in need of repair.

Art. 6. Recommendations.

In this article, the condition of the bridge, together with the efficiency of its members, will be discussed, and recommendations stated.

Floor System: The specifications call for a consideration of both uniform and concentrated line loadings, but owing to local conditions, which are treated in the next paragraph, it is not deemed necessary to consider concentrated loads. There is a 4 in. gas pipe running along the south side of the bridge, but its effect upon the loading is inappreciable, and will not be considered.

The city authorities, recognizing the fact that the bridge is of doubtful safety, have caused a sign to be placed at each end of

tire bridge, stating that only four teams will be allowed on the bridge at one time. That these restrictions are not strictly observed is evident, but they do prevent the passage of heavy engines, and the congestion of traffic which is necessary to produce a loading equivalent to the concentrated loading given in Art. 38 of the specifications.

The efficiencies of 0.6, and 0.83 of the joists and floor-beams, respectively, are too low to be accepted even if the bridge were new. The floor-beams are very badly corroded while the joists are in a dangerous condition. Where the flooring rests upon the joists, the joists have decayed to a depth of about 2 in., and over the floorbeams they have been crushed until they have loosened out, in many cases, as much as half their depth. The flooring, too, is, in many places, rotten and loose, but apparently more attention has been given to it than to the joists. Such conditions as these demand immediate attention, and, unless remedied at once, a serious accident is likely to occur.

Whipple Truss:

From Fig. A, it will be seen that, except for the upper chord, the sections of the members of the truss are apparently efficient. This, however, is based upon the assumption that the full original area of the section is effective, which assumption is in error, especially in the case of the upper chords. Fine material has sifted through the floor of the bridge, and has accumulated upon the

upper chord to a depth of several inches. Upon the removal of this debris, it was found that the cover plates and rivets were badly corroded, the heads of many of the rivets being almost corroded away. The corrosion of the other members is not so strikingly apparent, but examination shows that the entire structure has been materially weakened by corrosion. It appears that the bridge has never been painted, except in scattered places where it seems to have been demanded on account of more extensive corrosion.

That the sections are weaker than the calculated efficiencies indicate, is shown by the fact that both rods of U3L5 of the north truss and L5U7 of the south truss, both members having the same section and calculated efficiencies of 1.19, are broken. This indicates that the actual efficiencies are at least 16% less than those calculated. According to this method of reasoning, all members of the lower chord are inefficient, except L4L6 which would have an efficiency of 1.01, the lowest efficiency being 0.865. The diagonal members are also inefficient, except those in the two center panels, the highest efficiency being 1.03 and the lowest being 0.89. The upper chord has a computed efficiency of 0.77, but on account of the extensive corrosion which has taken place in that member, it is considerably weaker than such an efficiency would indicate.

The failure does not seem to have been upward, as shown in Fig. 9.

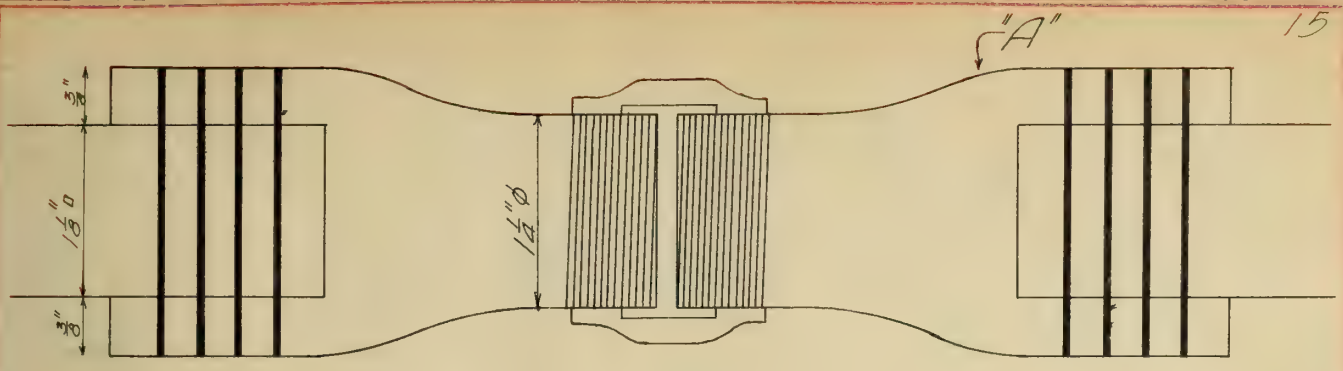


Fig. 9.

It will be noted that the effective area of the repair is only 97% of the original area. The clevis on one of the connecting rods is broken on one shoulder, at "A" Fig. 2. This practically destroys the usefulness of that member.

The above reasoning as to the reliability of the computed efficiencies applies with equal force to the lateral systems. Applying this rule to the top lateral system, Fig. 5(c), the efficiency in the end panel reduces to 0.924 while the rest are above 1.0. The top lateral struts are not considered as they are floor-beams and are treated under the floor system. The lower lateral system, Fig. 5(d), is not seriously affected by the above reasoning, but it exhibits two striking examples of bad practice. The most serious of these is the manner in which the diagonals are connected. As shown in Fig. 10, an eccentric stress is developed at "B". This has proved disastrous at L_4 of the north truss, where the member has broken, and the free end has been bent around the lattice bars of the lateral strut. The other ex-

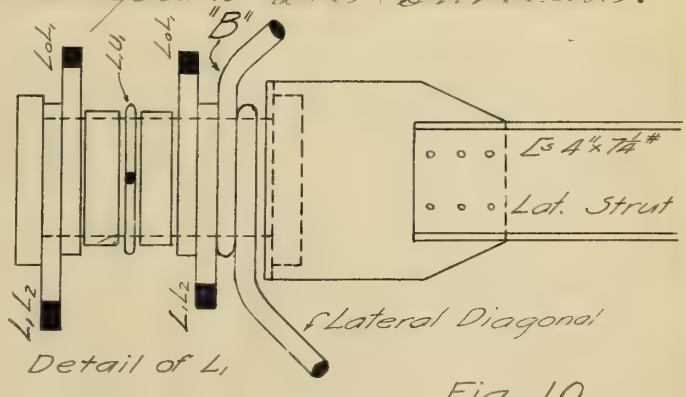


Fig. 10.

angle is the connections of the struts, the details of which are also shown in fig. 10. It is readily seen that unless the packing is sufficient to prevent any motion of the U plate along the pin, the member will be incapable of resisting compression until a certain amount of deformation shall have taken place in the lower chord. It is very difficult, if not impossible, to secure sufficient packing, and therefore, such a connection cannot be relied upon.

The sway bracing is satisfactory, and needs no further discussion.

It might be well to compare the specifications used in this investigation with other standard specifications in regard to the truss loading. For the span under consideration, the loading given in Waddell's specifications is 96 lb. per sq. ft., and that given in Ketchum's specifications is 85 lb. per sq. ft., while an Illinois statute requires that all bridges be designed to sustain a live load of 100 lb. per sq. ft. Hence it appears that Cooper's loading of 65 lb. per sq. ft. is considerable below the average, which fact applied to the bridge under discussion tends to render it even less efficient.

Fifth Truss:

As in Art. 1, only one truss will be discussed. By referring to fig. 6, (a), it will be seen that, with the exception of $U_1 U_2$, all of the members in the truss are apparently efficient. The reasoning in regard to compression members in the discussion of the Whipple truss applies with

cannot have to this truss, as it is subject to the same conditions and influences. The application of this method of reasoning to the web members reduces the efficiencies 16%, giving a minimum of 0.84. The upper chord has a computed efficiency of 0.77 but on account of the extensive corrosion which has taken place in those members, they are considerably weaker than such an efficiency would indicate.

In regard to the top lateral systems, a 16% reduction of the efficiencies gives (61%) an efficiency of 1.66. It follows that the remaining diagonals have higher efficiencies. The struts are floor-beams, and were treated in connection with the floor system. The members of the lower lateral system are efficient, but the struts noted in the bottom lateral system of the middle truss, and shown in Fig. 10, are present.

The sway bracing is efficient.

Trestle Bents:

A casual glance at the columns of the trestle bents impresses one with the idea that they are entirely too light for the structure, and this is amply borne out in the efficiencies shown in Figs. 7 and 8. These efficiencies are materially reduced by corrosion making the condition of the bridge even more precarious. These members require no further discussion.

Foundations:

The foundations are treated fully in Art. 5, and do not require further discussion.

Conclusions:

In view of the foregoing investigation and discussion, the bridge is obviously unsafe in all of its vital parts, and it is recommended that the entire structure be torn down, and replaced by a modern bridge of class A1.

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